BENCHMARKING OF COMMERCIAL SOFTWARE FOR THE SEISMIC ASSESSMENT OF MASONRY BUILDINGS

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SUMMARY

In the present work a comparative study on the evaluation of the seismic response prediction of two buildings was made, using two Italian computer codes based on macro-elements and pushover analysis, seeking to gather knowledge on the needs for national applications. The buildings response to the earthquakes predicted by the two programs, characterized by the base shear, the deformation capacity and also the maximum ground acceleration supported was compared. The results obtained show the good performance of the methods based on modelling by macro-elements, which provide realistic predictions of the structure response to the earthquake with regard the base shear. In some cases good agreement is also found in terms of deformation capacity. In correspondence to the modelling by macro-elements, the non-linear static analysis used by the two computer codes evaluated seems to be a good and easily understandable approach.

1. INTRODUCTION

Masonry is one of the older constructive systems, and also one the systems closer to nature. In a wealthy past numerous palaces and temples were built in Portugal, many of whom collapsed due to the 1755 Lisbon Earthquake (Figure 1a). With the exception of the “pombalino” anti-seismic decree, Portugal was not a country concerned about the seismic safety of their buildings. The first way to prevent the effect of earthquakes was, in the absence of design tools, the obligation to adopt anti-seismic systems for the buildings (Figure 1b). The accumulation of knowledge about the earthquake action and their effects, associated with the computer systems development, allowed, on one hand, the safety evaluation of the built heritage, and on the other hand, the design of new structures, based on modern aspects of safety and structural design. In Portugal, the structural assessment of historic buildings has received much attention, but no advanced commercial software for masonry structures is readily available in the market.


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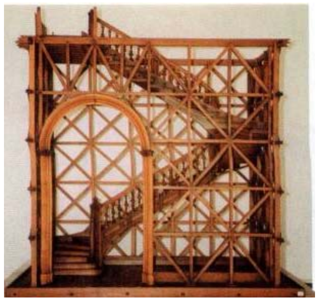

(b)

Figure 1: Consequences of the Lisbon Earthquake: (a) destruction of the city; (b) a building system with “pombalino” cage

If in Portugal the lack of commercial software for structural masonry is a consequence of lack of use of this typology, in other countries, by contrast, the evolution of masonry structures in parallel with reinforced concrete and steel structures has allowed the development of specific computer tools for masonry. Thus, the benchmarking process of tools developed on these countries (Figure 2), which incorporate also laws and assumptions established from experience, seems to be the wisest form to develop tools for the evaluation of the

Portuguese structures, adding them the national knowledge and adapting them to the specificities of the national buildings, such as the materials and the construction practices. For non-experts, benchmarking is seen as a positive and pro-active process by which a company examines how others perform a specific function in order to improve the way of doing the same or a similar function.

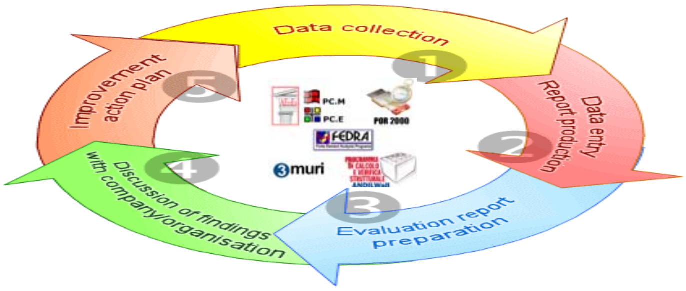


Figure 2: Benchmarking process over computer codes for structural masonry

In a previous study (Maciel, 2007) a survey of the software on the market for structural masonry has been done, as shown in table 1. While some of these tools were developed in cooperation with universities and research centres, others were developed only in a commercial environment, and therefore different approaches and performances are expected. Furthermore, the tools identified are from different countries, which have different realities in terms of resources and codes for masonry, and different building technologies.

Table 1 – Commercial computer codes for structural masonry (Maciel, 2007)

Program	Country	Code	Website
AEDES	Italy	Italian	www.aedes.it
CMT+L	Spain	Eurocodes	www.arktec.com/cmtl.htm
FEDRA	Norway	Eurocodes	www.runet-software.com/FEDRA.htm
WIN-Statik MurDim+	Sweden	Unknown	www.strusoft.com
Por 2000	Italy	Italian	www.newsoft-eng.it/Por2000.htm
TQS CAD/Alvest	Brazil	Brazilian	www.tqs.com.br/v13/alvest.htm
Tricalc.13	Spain	Eurocodes	www.arktec.com/new_t13.htm
Tricalc.17	Spain	Eurocodes	www.arktec.com/new_t17.htm
WinMason	USA	American	www.archonengineering.com/winmason.html
3Muri	Italy	Italian	www.stadata.com
ANDILWall	Italy	Italian	www.crsoft.it/andilwall
MURATS	Italy	Italian	www.softwareparadiso.it/murats.htm
Sismur	Italy	Italian	www.franiac.it/sismur.html
TRAVILOG	Italy	Italian	www.logical.it/software_travilog.aspx
Tecnobit	Italy	Italian	www.tecnobit.info/products/murature.php
CDMaWin	Italy	Italian	www.stsweb.net/STSWeb/ITA/homepage.htm

Maciel (2007) made an evaluation of the AEDES, FEDRA and Por 2000 programs. This evaluation is now extended to the ANDILWall/SAM and 3Muri Italian computer codes, doing a comparison of results between the two programs in the seismic analysis, performed over two buildings.

2. MODELLING AND SAFETY VERIFICATION

For a wall with openings it is possible to make a division in macro-elements (left of Figure 3), identifying the masonry panels located in horizontal (pier panels) and vertical (spandrel panels) alignments of the opening and the elements that make the connection (cross panels) between vertical and horizontal panels. While the pier panels have essentially a bearing function, the spandrels transmit the floor loads to the cross panels. In addition, the cross panels transfer the slab loads to the pier panels and ensure the connection between piers and spandrels (Augenti and Romano, 2008). Another important aspect in the geometrical discretization is the definition of the height of the pier panels, which is shown in the right of Figure 3 as proposed by Dolce (1989) and Augenti (2004).

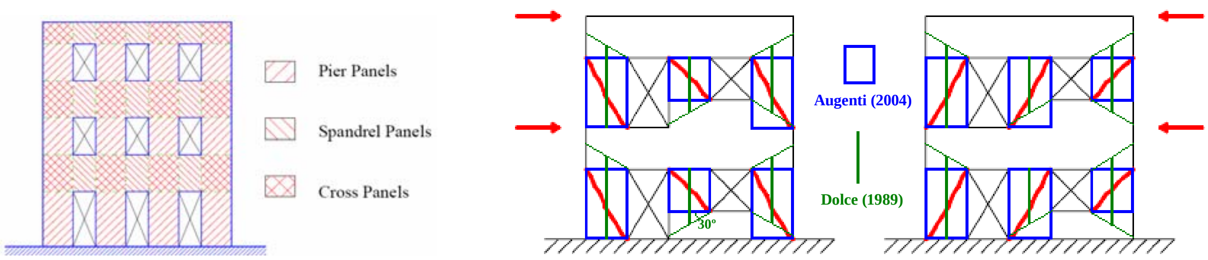
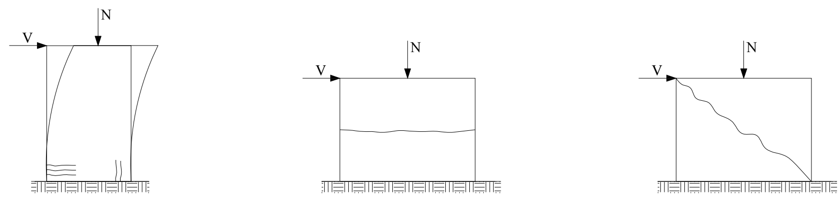


Figure 3: Wall division in macro-elements (left) and definition of the height of pier panels (right)

The behaviour in the plane of the walls is usually considered crucial to the response of the building, provided adequate box behaviour is ensures by the floors of extra ties. On a wall with openings the masonry piers are the elements with more influence in the strength of the structure, because they absorb directly the horizontal seismic action and they receive the vertical loading transmitted by the spandrels. In the panels loaded horizontally in the plane three types of collapse mechanisms are usually identified, including combined flexural failure and collapses by shear involving sliding or diagonal cracking (figure 4). The spandrels have, however, a considerable importance, because their characteristics of stiffness and resistance affect the behaviour of each wall, and therefore of the entire building.



(a) combined flexural (b) sliding (c) diagonal

Figure 4: Collapse mechanisms

For a correct simulation of the masonry panels mechanism and their interactive behaviour different types of macro-elements have been proposed, as the formulations proposed by Gambarotta and Lagomarsino (1996) and Magenes and Calvi (1996) shown in Figure 5, which are incorporated in the 3Muri and SAM computer codes, respectively. While the 3Muri formulation is based on the kinematic equilibrium of the macro-elements according to the degrees of freedom, the SAM creates an equivalent frame for a global analysis.

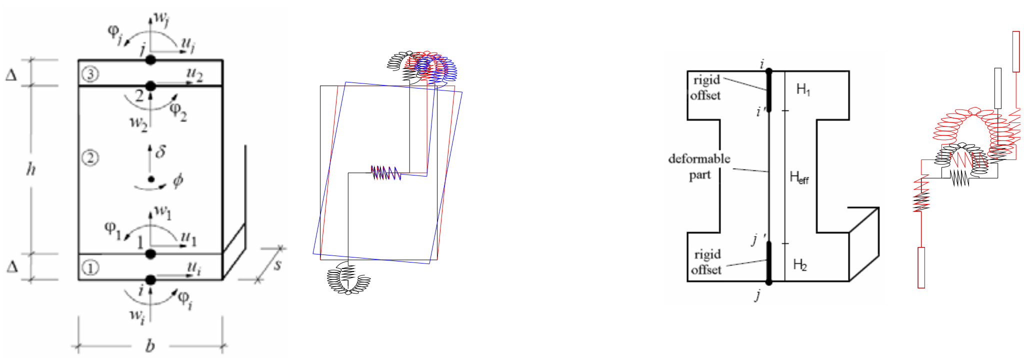


Figure 5: Macro-element and respective springs model in the 3Muri (left) and in the SAM (right)

It is known that the traditional elastic safety verification regarding the earthquake action does not consider the contribution of the masonry spandrels and the correct boundary conditions. Thus, the non-linear static analysis (pushover) appears as a preferable methodology in the safety verification, which allows exploring the ductility of the structure. The non-linear static analysis is the only method that can simulate the evolution of the condition of the structure during the earthquake, through the application of incremental horizontal forces until collapse. The behaviour of the structure is represented by the so-called “capacity curve” which represents the value of the base shear (horizontal force representative of the seismic action) versus the displacement of a control point (significant point of the structure, usually corresponding to the mass centroid of the roof slab).

According to Magenes (2006) the experience of past earthquakes shows that the performance assessment or safety check, and the consequent structural model, should consider the so-called “first damage mode” mechanisms, which involve usually out-of-plane damage and collapse mechanisms, and the “second mode” mechanisms, which are associated to in-plane response of walls. During an earthquake both out-of-plane and in-plane response are simultaneously mobilized, but it is generally recognized that a satisfactory seismic behaviour is attained only if out-of-plane collapse is prevented and in-plane strength and deformation capacity of walls can be fully exploited (Figure 6). However, the assumption that the resistance of the building to horizontal actions is provided by the combined effect of floor diaphragms and in-plane response of structural walls seems to be a good approach for a pushover analysis.

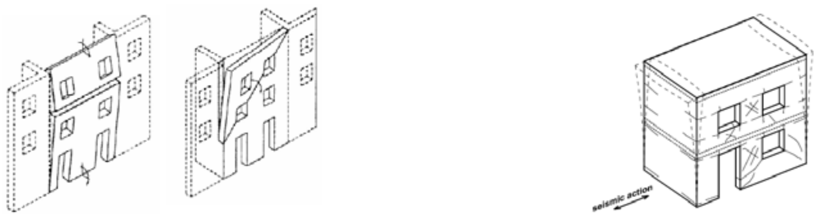


Figure 6: Examples of out-of-plane damage mechanisms (left) and global response mechanism (right) (Magenes, 2006)

In this study the floors are assumed as rigid diaphragms, which can be questionable in old structures, unless they include ties or well-connected floors. In the absence of rigid diaphragms, limit analysis of local failure mechanisms with macro-elements may be more adequate.

Figure 7 shows the safety verification process by a pushover analysis, modelling the building through macro-elements. Given that the assumptions made in the modelling of a building and the strategies adopted for its structural viability may require various analysis cycles the modelling with macro-elements presents the benefit of alleviating the high computational effort normally associated to the finite element method.

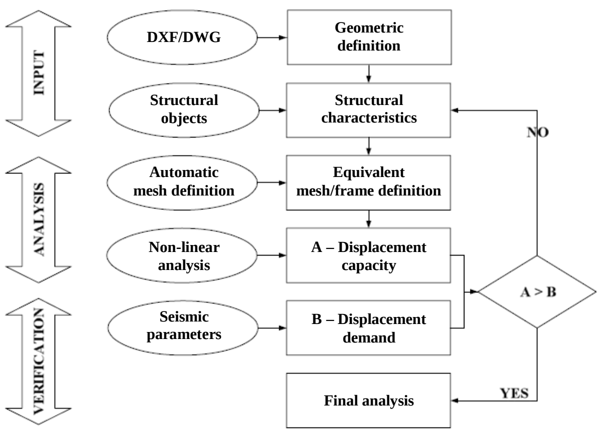


Figure 7: Seismic verification by non-linear analysis (S.T.A. DATA, 2007)

3. CASE STUDY

3.1. Building A

The first building analysed (Figure 8) was called “building A” and presents two floors with the plan dimensions 5.00 x 4.00 m^2 and a height of 3.00 m. Each floor presents only two openings in X-direction: a door with 1.00 x 2.00 m^2 and a window with 1.00 x 1.00 m^2. The walls have a thickness of 0.25 m, and a reinforced concrete slab with 20 cm thickness covers each floor. The architectural simplicity of this building has been chosen to capture the essence of the analysis methods.

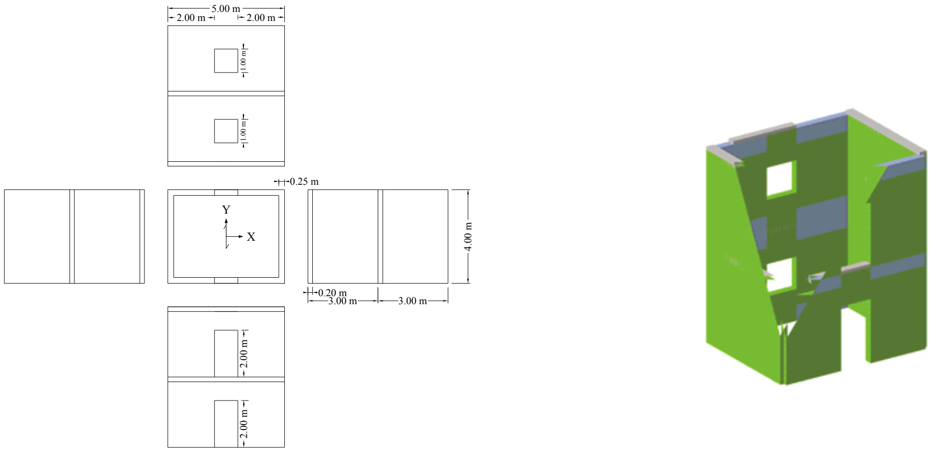


Figure 8: Building A: plan and views (left), and 3D mesh generated by 3Muri (right)

3.2. Building B

The second analysed building (Figure 9), “building B”, consists of two blocks with the plan dimensions 4.00 x 5.00 and 5.00 x 6.00 m^2. The building has two floors with a floor height of 3.00 m, and features two windows of 1.00 x 1.00 m^2 in X-direction and three doors of 1.00 x 2.00 m^2 in Y-direction. The walls have a thickness of 0.25 m, and a reinforced concrete slab with a thickness of 20 cm covers each floor. The geometry of this building has been defined in order to check how the programs tackle the case of intersection of different wall plans.

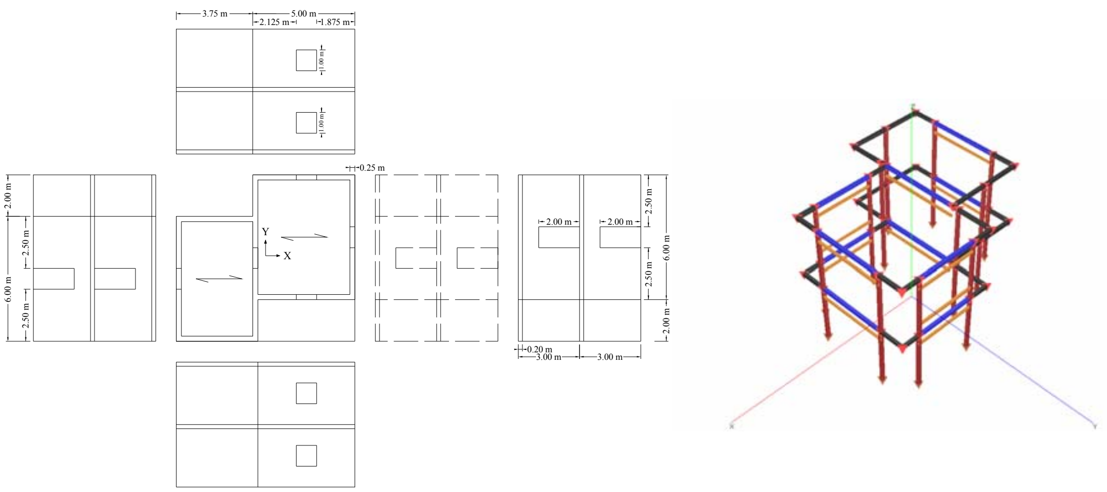


Figure 9: Building B: plan and views (left), and 3D frame generated by SAM (right)

3.3. Materials and loads

The buildings were modelled assuming the properties for masonry shown in table 2.

Table 2 – Masonry properties

UNITS	Type	Hollow concrete blocks
	Compressive strength, f_b	20 MPa
MORTAR	Type according to Eurocode 6	M4
MASONRY	Specific weight, γ	20 kN/m^3
	Compressive characteristic strength, f_k	5 MPa
	Pure shear characteristic strength, f_{vk0}	0.15 MPa
	Normal elasticity module, E	5 GPa
	Tangential elasticity module, G	2 GPa

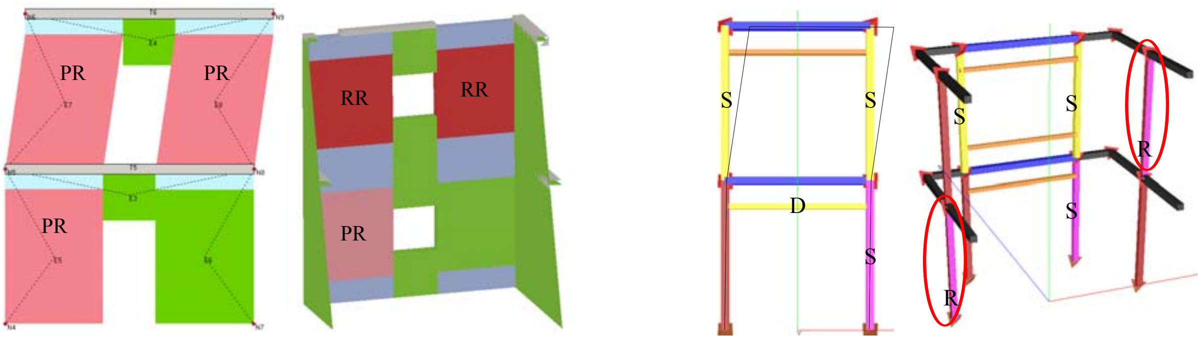
For building A the slabs were subjected to a dead load of 10 kN/m^2, while for the second building a dead load of 5 kN/m^2 was assumed.

4. RESULTS

4.1. Building A

The pushover analysis allows generalizing the global response of a building through the representation of the capacity curve. However, the identification of zones susceptible to local damage, particularly to mechanisms out-of-plane, is very important. The evaluated programs identify such zones and mechanisms, even if they are not concordant in both programs.

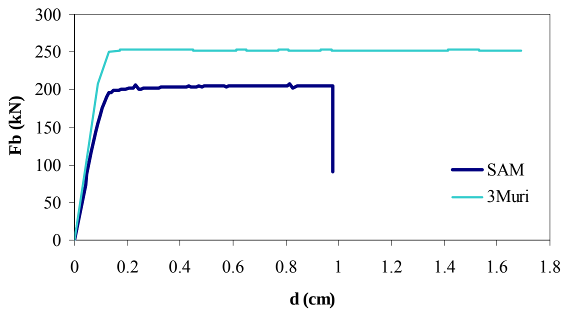
Figure 10a highlights the rocking mechanism found on the second floor by 3Muri, while Figure 10b shows also failure in the second floor, but this time by shear, according to SAM. While 3Muri does not detect any out-of-plane mechanism, SAM detects failure by combined bending in the walls indicated at right of Figure 10b.



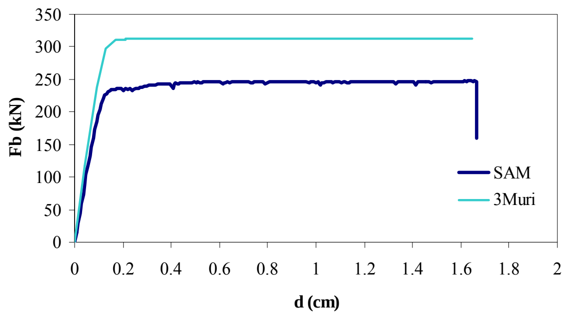
(a) (b)

Figure 10: Damage mechanisms for an earthquake in +X-direction with triangular distribution of forces: (a) from 3Muri where PR is plastic by rocking and RR is rupture by rocking; (b) from SAM where S is sliding shear and D is diagonal shear

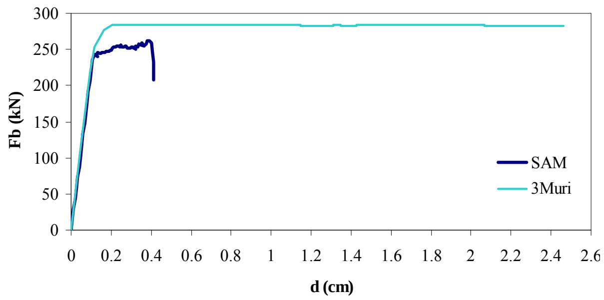
Figure 11 shows a comparison between the capacity curves obtained by the two programs in the +X and +Y directions, for triangular and uniform distributions of the equivalent static lateral forces to the seismic action, without consideration of accidental eccentricity.



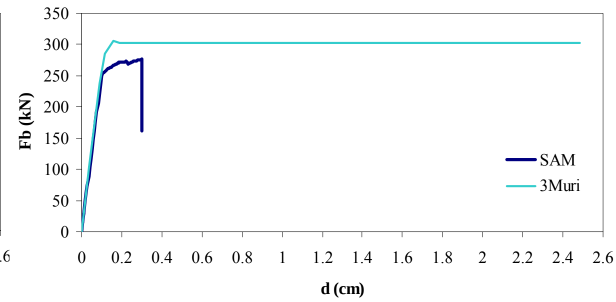
(a) +X earthquake with triangular distribution



(b) +X earthquake with uniform distribution



(c) +Y earthquake with triangular distribution

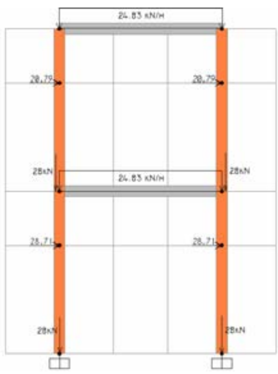


(d) +Y earthquake with uniform distribution

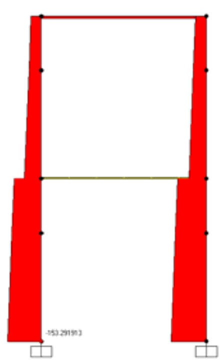
Figure 11: Comparison of the capacity curves calculated by 3Muri and SAM

The comparison of the capacity curves allows, in general, to conclude that the maximum base shear predicted by the two programs is not too distant and the same for the stiffness, particularly in +Y-direction. In the case of the program 3Muri, note that the maximum base shear is higher (10 to 25%) to that achieved with the SAM, which occurs because the 3Muri detects a rupture by flexure while SAM predicts collapse by shear.

To clarify the value obtained for the maximum base shear with the two programs, a calculation was made for the seismic action in +X-direction with a uniform distribution of forces, using the SAP2000 computer code (CSI, 2004). In this program the wall in X-direction was modelled with two openings, using frame elements (Figure 12a). A frame equivalent section in terms of axial, bending and shear stiffness was calculated, and these pseudo-walls were connected by horizontal frames with the section and mechanical properties of the border beams used in the modelling with the masonry programs. The masonry "frames" were considered with a weight equivalent to their walls, while the vertical load on the slabs was linearly distributed along the horizontal frames.



(a)



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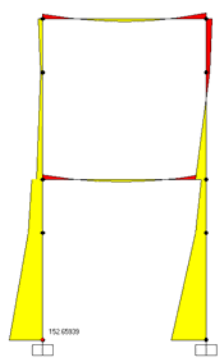


(c)

Figure 12: Calculation of the maximum base shear in +X-direction using the SAP2000 program: (a) modelling of the typical wall; (b) axial force diagram, N; (c) bending moment diagram, M

After modelling the equivalent wall, the distribution of the equivalent static lateral forces to the seismic action by the wall elements has been made, applying forces at the lower level of the spandrels. For a distribution proportional to the mass, with 58% of the weight on the first level and the remaining in the second level, after several attempts, it was possible to find the base shear of the wall (99 kN) that allows the most penalized element in terms of eccentricity not to fall outside of the section, i.e. M/N < L/2, where L is the length of the wall panel. The base shear so determined is valid for a two degrees of freedom system. To transform this force into an equivalent force for a single-degree-of-freedom system, it is necessary to divide this by the participation coefficient (O.P.C.M. n. 3431; Eurocode 8), calculated for the present case by Maciel (2007) as 0.80. Dividing the total base shear in +X-direction for the two degrees of freedom system (2 x 99 kN) by the participation coefficient a base shear of 247.5 kN was obtained, which approximates the value of the maximum base shear obtained from the SAM computer code.

With respect to the deformation capacity, there is agreement among the programs only in the case of the seismic action in +X-direction with uniform distribution of forces. The contrast is observed in +Y-direction, whit a ratio of 1:8 between the maximum displacements predicted by SAM and 3Muri.

4.2. Building B

Figure 13 shows the prediction of the deformation response of the building and of the damage mechanisms with respect to an earthquake in the +Y-direction with triangular distribution of forces. From this figure it is possible conclude that the deformation modes predicted by the two programs are identical, with displacement almost exclusive of the second floor. In terms of damage, the programs agree to detect the failure of all walls of the second floor, although the 3Muri identifies collapse by rocking and the SAM detects collapse by sliding shear.

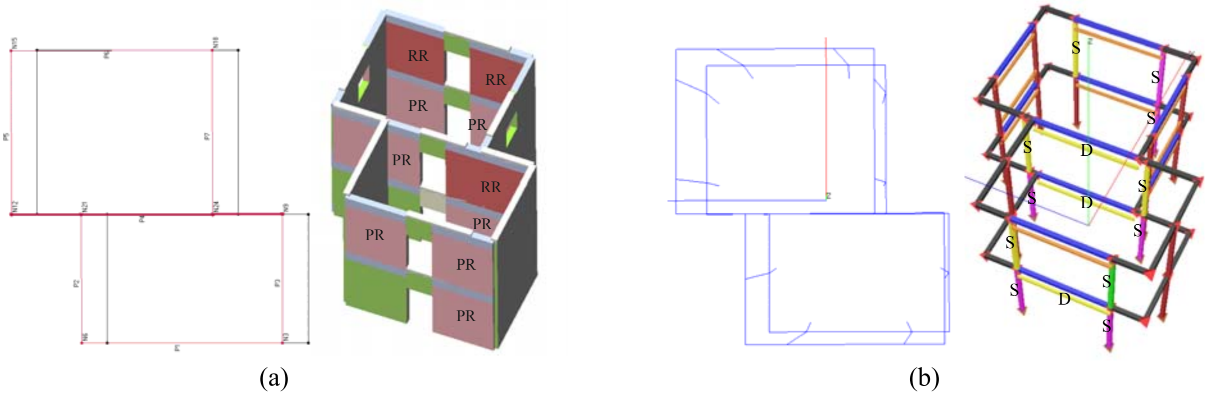
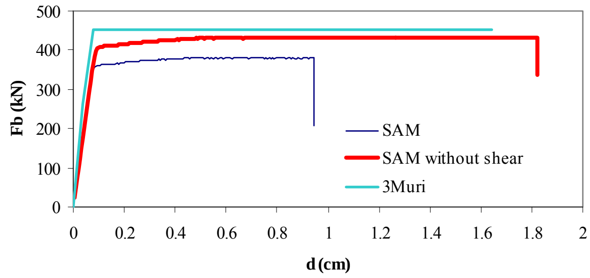


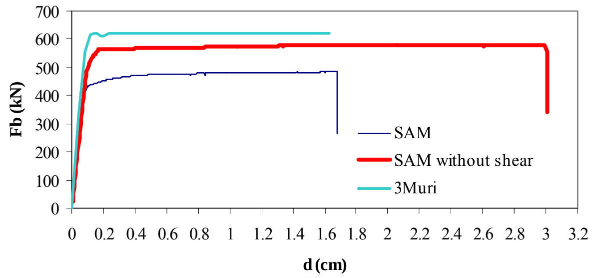
Figure 13: Deformed shape and in-plane damage mechanisms for an earthquake in the +Y-direction with triangular distribution of forces: (a) from 3Muri where PR is plastic by rocking and RR is rupture by rocking; (b) from SAM where S is sliding shear and D is diagonal shear

Figure 14 illustrates the comparison of capacity curves obtained with the methods SAM and 3Muri. The disagreement about the mechanism of rupture dictates the difference in the capacity curve, on the maximum base shear and with respect to the capacity deformation. Both quantities are smaller in SAM because are associated with a mechanism (sliding shear) typically more brittle.

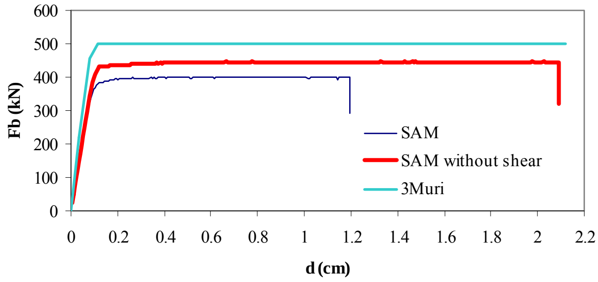
To establish a more effective comparison between the capacity curves obtained from the two programs, the model “SAM without shear” has been created, which is free of rupture by shear. The calculated capacity curves with the models SAM without shear and 3Muri show a good approximation in terms of base shear, in general, and for the deformation capacity in the case of the triangular distribution of forces.



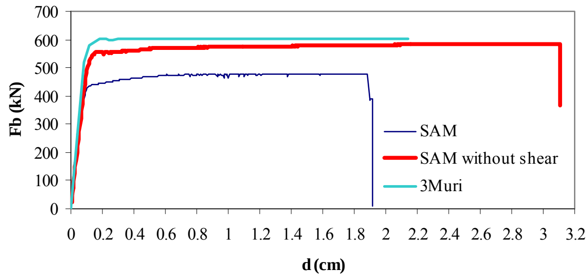
(a) +X earthquake with triangular distribution



(b) +X earthquake with uniform distribution



(c) +Y earthquake with triangular distribution



(d) +Y earthquake with uniform distribution

Figure 14: Comparison of capacity curves obtained with different models

For this building, a comparative analysis of the ground accelerations (ag) supported by the structure was also made, so that the different safety criteria adopted by the Italian code O.P.C.M. n. 3431 are satisfied, including the Damage Limit State (DLS), the condition of $q^* < 3$, and the Ultimate Limit State (ULS).

The q* parameter indicates the ratio between the elastic response force and the yield force of the equivalent system, and its control represents a limitation to the ductility of the structural system as a whole. In this study, we considered the following parameters for the seismic action: ground foundation type A according the Italian code O.P.C.M. n. 3431 and Eurocode 8, and the spectral parameters T_B, T_C and T_D defined in those codes, with values of 0.16, 0.40 and 2.40 seconds, respectively.

Table 3 presents the ground accelerations that constitute the upper limit for the verification of the various safety criteria. For the SAM program two calculations were made, with and without possibility of collapse by shear.

Table 3 – Limit ground accelerations for verification of the various safety criteria

		3Muri	SAM	SAM **
		$ag_{máx}$ (m/s^2)		
Criterion	DLS	2.70	4.22	4.32
	$q^* = 3$	6.50	5.59	6.38
	ULS	11.45	6.18	10.79

** Without shear mechanism

Table 4 shows the comparison of the maximum values of ag supported by the building considering only the analysis in +X-direction, with triangular distribution of forces. Table 5 shows a similar calculation, but considering the seismic action in +Y-direction, also with triangular distribution of forces.

Finally, table 6 identifies the weak direction of the building, i.e. the direction for which the seismic action is more severe. It also identifies the distribution of forces associated with the seismic action.

Table 4 – Limit ground accelerations for the +X earthquake with triangular distribution

		3Muri	SAM	SAM **
		$ag_{máx}$ (m/s^2)		
Criterion	DLS	2.70	4.61	6.57
	q* = 3	6.50	5.59	6.38
	ULS	11.52	6.18	10.79

** Without shear mechanism

Table 5 – Limit ground accelerations for the +Y earthquake with triangular distribution

		3Muri	SAM	SAM **
		$ag_{máx}$ (m/s^2)		
Criterion	DLS	3.16	6.47	5.40
	q* = 3	7.05	5.79	6.57
	ULS	13.54	7.06	11.77

** Without shear mechanism

Table 6 – Identification of the weak direction of the building / type of distribution of forces

		3Muri	SAM	SAM **
Criterion	DLS	+X / T	-Y / T	-Y / U
	q* = 3	+X / T	+X / T	+X / T
	ULS	-X / T	+X / T	+X / T

** Without shear mechanism; T: Triangular, U: Uniform

For the programs 3Muri and SAM, the criterion more consensual in terms of the limit ground acceleration appears to be the condition of q* < 3. It also seems clear that the +X-direction is crucial in the seismic safety verification of the building, which is associated in general to the triangular distribution of forces.

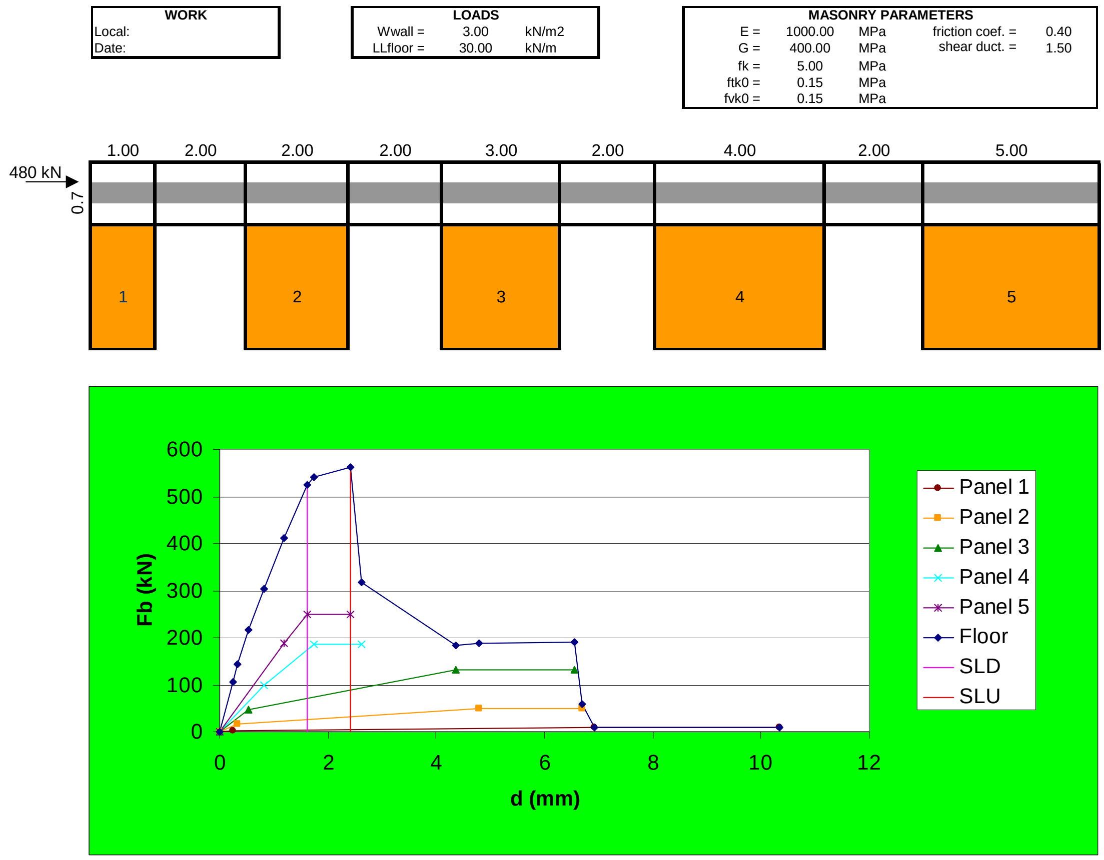
4.3. Conclusions

Overall, the two programs evaluated (SAM and 3Muri), even implementing a similar strategy for the seismic analysis, do not coincide in the definition of the collapse mechanism. This dictates a different response of the building. However, assuming a rupture by rocking, closed results were found. The difference are probably due to the different formulations of the macro-elements adopted and only further analysis of the formulations will allow a full characterization of this issue.

Regarding the capacity in terms of the maximum peak ground acceleration, there is better agreement between the values obtained with the two programs, particularly when the shear mechanism is not considered. The criterion that determines closer results between these values is the one that establishes the condition q* < 3.

The two programs evaluated in this study, although “commercial”, allow the definition of assumptions for the behaviour of the masonry and for the modelling of the structure, thus allowing a good coverage of structural typologies. Moreover, the economy of computer resources is evident with respect to tools based on the finite element method.

As a result of the benchmarking process conducted, a Portuguese simple software to evaluate the performance of masonry buildings is being developed (Figure 15). The approach adopted is based on the RAN method (Augenti, 2004), with implementation in a spreadsheet where the user needs to define the data for each wall of the building (geometry, loading, masonry properties). Afterwars, the calculation is done through the conditional sum of the various panels that form the wall, and the safety verification may be based on a criterion of force or displacement.



		displacement demand (mm)		displacement capacity (mm)	
		Spectra 1	Spectra 2		
SLD	$1.5\ S_{De}(T^*)/2.5$	0.481	1.099	1.444	OK
SLU	$1.5\ S_{De}(T^*)$	0.802	1.832	2.372	OK
	Ductility condition: $q^* < 3.0$				
	$1.5\ S_e(T^*) \times m^*/F_y^*$	0.238	0.543		OK

Figure 15: Program prototype to evaluate performance of masonry buildings

5. REMARKS

The development of design and assessment tools is necessary for the correct safety evaluation of masonry building heritage. It is also needed to standardize the grouns for the calculation of these structures, by defining valid hypotheses for its modelling and behaviour, which allow simulating the real response of the buildings with respect to the applicable loads, particularly to the seismic action. The state of the art and the results obtained in this study, show the good performance of the methods based on modelling by macro-elements, which provide realistic estimates of the performance of the structure with respect to the earthquake action, as regards the base shear supported, and even there is in some cases a good agreement in terms of the deformation capacity.

Together with the modelling by macro-elements, the non-linear static analysis seems to be a good and easily understood approach, because it is based on the simple evaluation of the requested deformation with respect to the displacement capacity of the building. This approach is also in correspondence with the quantification of seismic action recommended by the Eurocode 8, considering the reserve of non-linear capacity of the structure.

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